GROUP DELTA CONSULTANTS

Certified DBE/MBE

Geotechnical Engineering
Geology
Hydrogeology
Coastal Engineering
Hydrology
Hydraulics
Environmental Engineering

Project No. I-181-03
January 22, 1999

BOYLE ENGINEERING CORPORATION
7807 Convoy Court, Suite 200
San Diego, California 92111

Attention: Mr. Clark Fernon, Project Manager

DRAFT TYPE SELECTION REPORT
VEHICULAR UNDERCROSSING AT STATION 87+96.601
MIDDLE SEGMENT, STATE ROUTE 56
SAN DIEGO, CALIFORNIA
11-SD-56-KP 3.3 TO 10.5, EA 172820

Gentlemen:

We are transmitting five copies of our preliminary geotechnical type selection report for the Vehicular Undercrossing at Station 87+96.601, on the Middle Segment of the proposed State Route 56 alignment in San Diego, California. Laboratory testing is currently underway. A final Type Selection Report will be issued at the completion of the laboratory testing. Based on our assessment of the site conditions, we do not anticipate significant changes in our conclusions.

We appreciate the opportunity to be a part of your design team for this project. If you have any questions or require additional information, please call.

Very truly yours,
GROUP DELTA CONSULTANTS, INC.

Kul

REGISTERED PROFESSIONAL ENGINEER
KUL CHUSHAN
No. GE 000144
Exp. 12/31/01
GEOTECHNICAL
STATE OF CALIFORNIA

Kul Bhushan, Ph.D., G.E.
President

Distribution:
Addressee (5)

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GROUP DELTA CONSULTANTS

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1.0 INTRODUCTION

1.1 Background

This Type Selection Report is based on a geotechnical investigation performed by Group Delta Consultants, Inc. (GDC) to provide recommendations for the foundation design of the Vehicular Undercrossing at Station 87+96.601. The bridge structure is part of the Middle Segment of the proposed State Route 56 (Ted Williams Freeway), extending from Rancho Penasquitos to Carmel Valley, in the City of San Diego, California (See Site Location Map, Figure 1).

The County and City of San Diego and California Department of Transportation (Caltrans) District 11, have authorized improvements of the Middle Segment of State Route 56. The development limits for the overall Route 56 improvement project extend from Interstate Highway 15 (Escondido Freeway) in Rancho Penasquitos to Interstate Highway 5 (San Diego Freeway) in Carmel Valley. The Middle Segment contains 7 proposed bridges, and extends from metric Station 45+13.527 on the west (in Carmel Valley) to metric Station 109+00 on the east (near Rancho Penasquitos).

Our understanding of the proposed project is based on the following drawings provided by Boyle Engineering: 1:2000 scale plan and profile entitled "SR-56 Selected Alignment," dated August 10, 1998, and "Planning Study" drawings for the proposed bridges dated 1-98 through 9-98.

1.2 Existing Facilities and Proposed Improvements

The site of Vehicular Undercrossing at Station 87+96.601 is located where the proposed SR-56 alignment will pass over the alignment of a future 2-lane roadway. This roadway will allow access across SR-56 to serve future subdivisions in this area. The proposed centerline of this roadway crosses the centerline of the proposed SR-56 alignment at metric Station 87+96.601. With the exception of some agricultural activity, the bridge site remains in a generally natural condition. The Planning Study

General Plan for the bridge and a Topographic Map of the bridge site are presented in Figures 2 and 3, respectively.

1.2.1 Proposed Bridge

The proposed improvements consist of Vehicular Undercrossing at Station 87+96.601, where Route 56 will pass over the future vehicular roadway. Centerline stationing and elevations at the intersection are as follows: SR-56 (STA. 87+96.601, El. 84.793 m), Vehicular Road (STA. 16+85.290, El. 77.5 m ±). The Undercrossing consists of a left and right bridge. Each structure will be a single-span, cast-in-place prestressed concrete box-girder structure supported by abutments on the east and west. Span length is currently planned at about 32.9 m for both bridges. Each bridge deck is planned at 12.77 m wide, with a clear space between bridges of 20.67 m. The alignment of the vehicular roadway will be skewed relative to the SR-56 alignment at about 0 degrees, 54 minutes. Abutment fill slopes are proposed at 1:1.5 (vertical to horizontal), with heights on the order of 6 to 7 m. Full slope paving is proposed at both abutments.

The bridge site is located on a natural southeast facing slope, and crosses a cut-fill transition of the SR-56 grading. A southwesterly flowing alluvial drainage exists at the base of the slope. The geometry of the cut-fill transition and natural slopes in this area results in different foundation support conditions at the abutments, as follows:

BRIDGE	ABUTMENT	FOUNDATION CONDITIONS
Left	Abutment 1	Foundation Elev. about 5-6 m below exist. grade
Left	Abutment 2	Foundation Elev. about 2-3 m below exist. grade
Right	Abutment 1	Foundation Elev. about 1-2 m below exist. grade
Right	Abutment 2	Foundation in fill, about 2-4 m above exist. grade

We anticipate that all abutments may be supported on spread footings founded either in the abutment fills or in dense formational materials. Only two borings were drilled for this Type Selection Study. Additional borings should be performed in order to determine appropriate foundation elevations and more precisely define support conditions at abutment locations that have not yet been drilled. Details of our preliminary foundation recommendations are presented in Section 4.1 of this report.

1.2.2 Design Foundation Sizes and Loads

No data is available on foundation loads at this time.

1.2.3 Existing and Proposed Cut/Fill Slopes

No existing cut or fill slopes are present at the site of the proposed bridge.

The slopes to be constructed below the abutments are proposed at a 1:1.5 (Vertical: Horizontal) gradient, and will be about 6 to 7 m in height. The configuration of abutment slopes is complicated, due to the cut-fill transition and canyon geometry in this area. The arrangement is described in general below:

- At Abutment1, Left Bridge, the entire abutment slope will be cut.
- At Abutment 1, Right Bridge, the abutment slope will consist of about 1.5 to 3 m of fill overlying cut.
- At Abutment 2, Left Bridge, the abutment slope will be about 1.5 to 2.5 m of fill overlying cut.
- At Abutment 2, Right Bridge, the abutment slope will be predominantly fill, with the exception of about 0 to 1.5 m of cut at the toe of the slope.

These complicated geometric conditions should be considered when planning the locations of additional borings at the bridge site.

The 1:1.5 slopes below the abutments will be paved with concrete. The slopes of mainline SR-56, both cuts and fills, will be unpaved, and are currently proposed at a gradient of 1:2.

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 Field Exploration Program

To investigate the subsurface conditions at the bridge site, two 20.3 cm diameter hollow-stem auger borings were drilled on January 11, 1999, to depths between 13.9 and 18.4 m below existing grade. The location of these borings are presented in Figure 3. Bulk and drive samples were taken during the drilling operation at selected depths for identification and laboratory testing. All drive samples were advanced with a 63.5 kg hammer dropped from a height of 76.2 cm. The sampler penetration resistance, or number of blows, to advance the sampler 30 cm was measured and recorded on the boring logs to assess the in-place density or consistency of the site soils.

Intact samples were obtained with a 6.15 cm I.D., 7.62 cm O.D., California Ring Drive Sampler. Representative samples were obtained from cuttings from the auger as well as a Standard Penetration Test (SPT) drive sampler. Samples were visually identified and classified in the field in accordance with the Unified Soil Classification System (USCS), placed in moisture tight containers, labeled, and taken to the laboratory for further inspection and testing. Pocket penetrometer tests were performed on cohesive ring samples. Boring logs are presented in Appendix A.

2.2 Laboratory Testing Program

Selected samples were tested in the laboratory to measure relevant engineering properties. Testing was performed in general accordance with applicable Caltrans testing methods, where appropriate. The following types of tests were performed:

- Moisture Content and Dry Density
- Grain-Size Distribution
- Liquid and Plastic Limits

- Direct Shear
- Corrosivity (pH, minimum resistivity, Sulfates, Chlorides)
- Pocket Penetrometer

The results of the laboratory tests are presented in Appendix B. (to be completed)

3.0 SITE AND SUBSURFACE CONDITIONS
3.1 Climatic Conditions

The project is located between Carmel Valley and Rancho Penasquitos in the City of San Diego, California. Site elevations range from approximately 70 to 98 m above mean sea level (MSL). The annual rainfall ranges from approximately 30 to 38 cm with over 95% of all precipitation occurring between October and May. The area has a semi-arid climate with average high temperatures during the year ranging from 15 to 21 degree C during the winter months to 27 to 32 degree C during the summer months. Average lows are generally 0 to 5 degree C during the winter months, to 10 to 15 degree C in the summer. Soil freeze/thaw conditions are not known to exist within the project alignment.

3.2 Subsurface Conditions
3.2.1 Geology and Soil Conditions

The project site lies within the Peninsular Ranges Geomorphic Province of California, in the coastal plain area of San Diego. The mesa topography of the coastal plain is characterized by low hills and ridges dissected by intervening alluvial canyon drainages. This area is generally underlain by terraced coastal sedimentary formations of Quaternary to Tertiary age. These formations are overlain locally by Holocene (recent) overburden deposits such as alluvium, slopewash, and man-placed fill soils.

The proposed bridges will span the future vehicular access road, a north-south trending 2-lane roadway. The roadway alignment traverses a southeasterly facing natural slope, as shown in Figure 3. Test borings indicate that this natural slope is underlain by about 0.6 to 1.8 m of overburden deposits, underlain by Eocene sedimentary formational material of the Torrey Sandstone (Tt). The geologic units encountered are described below.

3.2.1.1 Overburden Deposits

Our test borings encountered overburden soils to depths of 0.6 to 1.8 m below existing grade on the natural slope underlying the bridge site. These soils include topsoil and residual clay. Topsoil at the site consists of loose to medium dense, moist, brown, silty and clayey medium to fine sands (SM, SC), some of which has been recently cultivated. Residual clay is a product of in-place weathering of formational soils, and is described as hard, moist, brown, sandy clay (CL). Equivalent Standard Penetration Test (SPT) blowcounts measured in the overburden soils range from 17 to 22.

3.2.1.2 Torrey Sandstone (Tt)

Torrey Sandstone was encountered in our test borings at depths between 0.6 and 1.8 m below existing grade, corresponding to El. 83.1 m at Boring UAR-HSA-1, and El. 77.1 m at UAR-HSA-2. This unit is characterized as very dense, moist, mottled light brown to light gray-brown with orange mottles, fine sand to silty sand (SP, SM), locally with weak to moderate cementation, and occasional thin interbeds of hard, moist, dark gray, sandy silt (ML). Equivalent SPT blowcounts measured within the Torrey Sandstone were generally greater than 100 blows per 0.3 meters.

3.2.2 Groundwater

The only groundwater in our two borings was a minor perched water zone at a depth of about 18.2 m in Boring UAR-HSA-1, corresponding to El. 66.7 m. It is possible that minor seeps may be encountered in foundation excavations where groundwater perches on less pervious soil layers and flows laterally through more pervious layers. Groundwater flow quantities typically vary seasonally due to variations in precipitation and surface infiltration.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Proposed Foundations

We anticipate that Abutments 1 and 2 of both bridges may be supported on spread footings founded either in fill or dense formational soils. As an alternate, drilled piles may be used where foundations are in fill, if the anticipated settlements are not acceptable. Footing elevations are estimated at about El. 80.5 m at Abutment 1 and El. 80.0 m at Abutment 2. Subsurface data suggests that both the Abutment 1 and Abutment 2 foundations of the Left Bridge will be founded in cut in dense formational soil. For the Right Bridge, the Abutment 1 footing could likely be supported on cut in dense formational soil, depending on the depth to formation in this area (a boring has not yet been performed in this area). Abutment 2 of the right bridge, however, will clearly be supported in fill. From a foundation standpoint, the formational soils where encountered at and below the proposed foundation elevations will provide good bearing support. It is our opinion that the abutment foundations can be supported on spread footings where formation is present, and can be supported either on spread footings or drilled piles where compacted fill is present.

For abutment footings adjacent to 1:1.5 slopes, the design should provide for a minimum setback of 2 m from the face of the slope. The bottom of footing should be embedded a minimum of 1.5 m below the slope face directly above the outside edge of footing to provide improved bearing, lateral, and uplift capacity. Minor dewatering of the footing excavations may be required during construction, if perched groundwater is found.

For footings founded in formational materials, the base of the excavation should be clean and free of loose debris, and the upper 0.15 m scarified and recompacted. An allowable net bearing pressure of 290 kPa may be used for footings founded in formational soil adjacent to 1:1.5 slopes, provided minimum setback and embedment criteria above are satisfied.

For abutment footings supported in compacted fill, the compacted fill placed within 1 m below the bottom of the footings should not contain materials larger than 76 mm across, and should be compacted to a minimum relative compaction of 95%. All fills under foundations or behind abutment walls should be compacted in accordance with Caltrans Standard Specifications for structural backfill.

For abutment footings supported in fill adjacent to 1:1.5 slopes, we recommend an allowable net bearing capacity of 215 kPa, assuming minimum embedment as described above. If additional bearing capacity is desired, the footings may be deepened. For each additional 0.3 m of embedment below the minimum of 1.5 m, the allowable bearing pressures may be increased by 25 kPa.

4.2 Settlement

Settlement of abutment footings is expected to occur rapidly, and the majority of settlement should occur shortly after application of the structural loads. Both abutments of the left bridge are anticipated to be in cut and expose formational soils at the foundation level. Estimated total settlement for these abutment is expected to be less than 1.3 cm.

Abutment 1 of the right bridge may be have up to 2 m of fill above the formational soil. Abutment 2 is anticipated to have 2 to 4 m of compacted fill above the formational soils. The estimated total settlement of the abutment under 4 m of fill is on the order of 3 cm. Differential settlement between the abutments is anticipated to be less than 1.3 cm.

4.3 Seismic Design Considerations

4.3.1 Ground Surface Rupture

The site is not located within the Alquist Priolo Fault zone. No faults were discovered on the site during our field investigation. Faults are not mapped as crossing the site or projecting towards the site in the geologic literature reviewed. As such, the possibility of ground rupture at the site is extremely remote.

4.3.2 Seismic Shaking

The site is located in a moderately-active seismic region of southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking due to nearby and distant earthquakes should be anticipated during the life of the facilities. The controlling fault for this project is the Rose Canyon Fault, located a distance of about 11 km from the site. The fault has a maximum credible earthquake magnitude of 7.0. Based on the Caltrans 1996 California Seismic Hazard Map, we recommend using a PGA of 0.3g for design. Depth to bedrock may be taken as 3 to 25 meters.

Response spectra at the bridge site should be selected in accordance with Applied Technology Council (ATC-32: Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, 1996) for soil profile Type C, with an applicable earthquake magnitude of 7.25 $\pm$0.25, and a PBA of 0.3 g (Figure R3-5 of ATC-32).

4.3.3 Secondary Seismic Effects

Secondary seismic effects for any site include liquefaction, seismic compaction, settlement, and slope instability.

Liquefaction involves a sudden loss in strength of a saturated, cohesionless soil (predominantly sand) caused by cyclic loading such as an earthquake. This results in temporary transformation of the soil to a fluid mass. Typically, liquefaction occurs in areas where groundwater is less than 9 m from the surface and where the soils are composed predominantly of poorly consolidated fine sands. Liquefaction could occur locally in the alluvium in the creek bed. However, since no groundwater is present and all foundations are to be supported in the dense formational soils or compacted fill and should not be affected by liquefaction. Before construction of the abutment fills, all fill should be properly keyed into formational soils to minimize any potential for lateral spreading of the abutment slopes.

Settlement of dry sands can be caused by the cyclic loading of an earthquake. A procedure for estimating the probable settlement of dry sands was developed by

Seed and Silver (1972). This procedure was reviewed by Tokimatsu and Seed (1987). Based on this procedure and the relative density of the soils at the project site, the settlement of dry sands at the site are not expected to be significant.

Slope instability, in the form of landslides and mudslides, is a potential adverse impact associated with seismic shaking. The proposed 1:1.5 cut and fill slopes at the abutments, if properly compacted, keyed at the toe, and benched into the formation materials, are anticipated to be stable under seismic shaking.

4.4 Excavation Characteristics

Based on drilling characteristics and our experience in the area, the formational soils underlying the site may be excavated with medium to heavy effort by conventional heavy-duty grading equipment. The planned excavations may encounter minor to moderate amounts of cemented concretions within the formational soils which may require localized heavy ripping effort.

4.5 Permanent Slopes

Paved cut and fill over cut slopes, about 6-7 m high, with a gradient of 1:1.5 are planned below the bridge abutments, while 1:2 unpaved slopes are planned for mainline Route 56 slopes. Paved slopes are anticipated to be grossly stable, if keyed at the toe and benched into the competent formational soils. Unpaved fill slopes will be subject to surficial erosion and rilling if subjected to heavy rainfall.

Planting of the unpaved slopes with appropriate, drought tolerant vegetation (using minimal irrigation) should be done as soon as possible after excavation to guard against surficial erosion. Care should be taken not to allow surface water to flow over the slope face in an uncontrolled manner.

4.6 Scour

The bridge will not be constructed on an existing creek and scour is not a potential concern.

5.0 REFERENCES

California Department of Conservation, Division of Mines and Geology, 1994, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, Special Publication 42.

Caltrans, 1995, Standard Specifications, Business, Transportation and Housing Agency.

Mualchin, L., 1996, California Seismic Hazard Map (1996) Based on Maximum Credible Earthquakes (MCE).

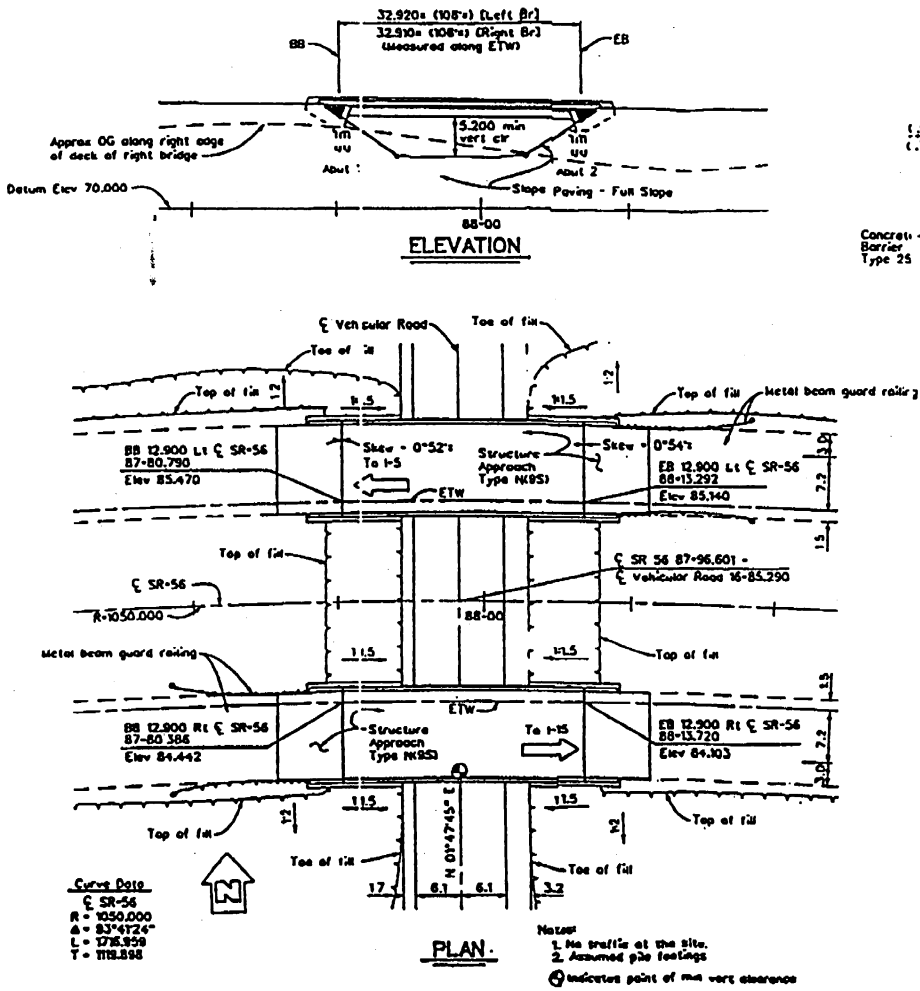
Seed, H.B. and Silver, M.L., 1972, Settlement of Dry Sands During Earthquakes, J. of Soil Mech. Found. Div., ASCE, Vol. 98, No. 4, pp. 381-397.

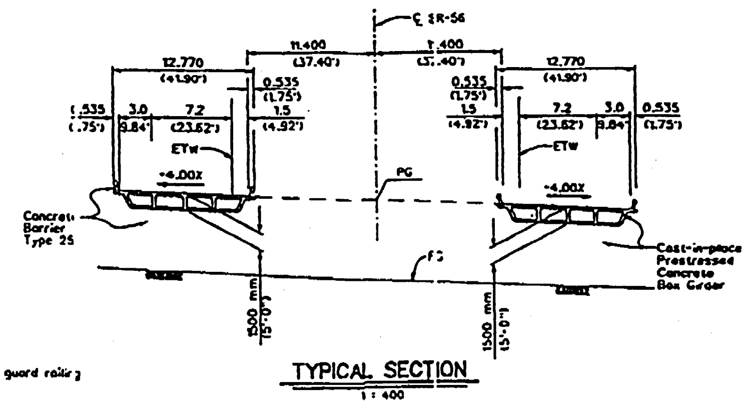
Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, J. of Geotech. Eng. Div., ASCE, Vol. 113, No. 8.

6.0 LIMITATIONS

The report, exploration logs, and other materials resulting from Group Delta's efforts were prepared exclusively for use in designing the proposed project. The report is not intended to be suitable for reuse on extensions, or modifications of the project, or for use on any other development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

Our recommendations and evaluations were performed using generally accepted engineering approaches and principles available at this time, and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is made.





Date of Estimate	12/98
Structure Depth	= 1500 m
Length	= 33 m
Width	= 12.770(2)=25.540 m
Area	= 843 sq m
Cost / sq m including 10% mobilization & 25% Contingency	= $ 998.30
(Cost / sq ft)	= $ 92.78
Total Cost	= $ 850,000

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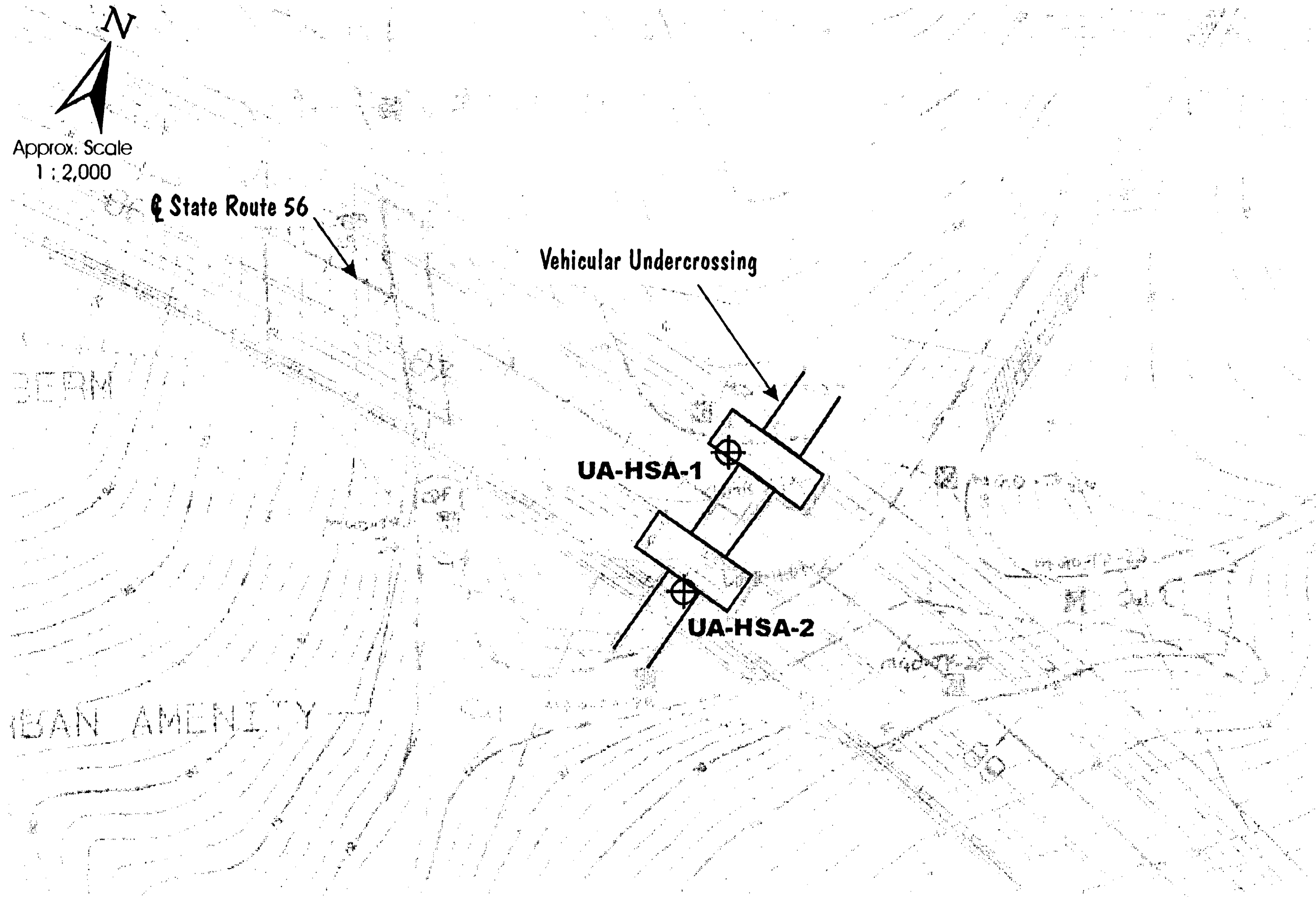
GROUP DELTA

GDC Project No. I-181

State Route 56
Middle Segment Bridges
Vehicular Undercrossing at
Station 87+96.60

GENERAL PLAN Figure 2



LEGEND

- ⊕ Hollow Stem Auger Boring
- Bucket Auger Boring
- ☒ Backhoe Test Pit

Reference:
The base map is from Boyle Engineering
SR-56 Selected Alignment, 8-10-1998

GROUP DELTA CONSULTANTS

GDC Project No. I-181
State Route 56
Middle Segment Bridges
Vehicular Undercrossing
EXPLORATION LOCATION MAP
Figure 3

BORING LOG

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 84.9 M	BORING NO.: UAR-HSA-1 B -
DRILL RIG: CME 85 (west Hazard)	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	

Depth	Sample #	Type	Blows ft	Recovery	DESCRIPTION		
	1	CA	15/6" 16/6" 17/6"	100% (1 set Ret)	loose to med dense, moist brn v. clayey med to fine sand (SC) Topsoil	22	
					Hard, moist brn sandy lean clay (CL) Residual clay		
					med dense, moist, Lt brn poorly graded fine sand w/ silt (SP-SM)		
5	2	SP	13/6" 15/6" 20/6"			30	
					V. dense, moist, Lt brn to Lt gray brn slightly silty fine sand (SM) w/ rare zones of brn to dark brn silty sand (SM) Formational		
	3	SAK					
10	4	CA	80/6"	100% (1 set Ret)		7100	
15	5	SP	38/6" 70/6"	100%		7100	
20	6	CA	86/6"	100% (1 set Ret)		7100	

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-18	8256-UAR	FIGURE NO.:

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B O R I N G   L O G   2 of 3

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 84.9 M	BORING NO.: UAR-HSA-1
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	B-

Depth	Sample #	Type	Blow Ct.	Recovery	DESCRIPTION			
	6	CA	86/6"	100% / 1st Ret		7100		
	7	SAK						
25	8	SP	25/6" 60/6"	100%		7100		
					cemented sand layer			
30					sample attempt at 30 ft 50 blows/no penetration sampler blocked by chunk of cemented sand			
					v. dense, moist, Lt grey w/ reddish brn mottles Silty med to fine sand (SM)			
35	9	SP	90/6"	100%	lightly cemented sand layer	7100		
					Torrey Sandstone?			
40	10	CA	165/6"	100% / 1st Ret		7100		

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-181	SR56-UAR	FIGURE NO.:

BORING LOG

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 84.9 ft	BORING NO.: WAR-HSA-1 B
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	

DESCRIPTION

Depth	Sample	Type	Blows	Recovery	Description		
	10	CA	120/6"	100% (1 set Ret)		>100	
						>100	
45	11	SP	65/6"	100%			
						>100	
50	12	CA	100/6"	100% (1 set Ret)			
						>100	
55	13	SP	150/6"	90%	cemented sand layer		
					cemented sand layer		
					WT est. @ 59.7ft	>100	
60	14	CA	80/6"	100% (1 set Ret)	BH. 60½'		

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-181	SR56-WAR	FIGURE NO.:

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BORING LOG

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 77.7M	BORING NO.: UAR-HJA-2
DRILL RIG: CME 85 (wet Hammer)	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	B.-

Depth	Sample #	Type	Blow ct.	Recovery	DESCRIPTION			
					~~loose, moist brn silty med to fine sand (SM) Road Fill~~			
	1	SP	7/6" 8/6" 9/6"	100%	med dense, moist brn silty med to fine sand (SM) w/ trace clay cultivated topsoil	17		
					V. dense, moist, Lt brn to Lt gray poorly graded med to fine sand w/ silt (SP-SM) Formational			
5	2	CA	45/6" 90/6"	80% (1st Ret)		7100		
	3	SAK						
10	4	SP	60/6"			7100		
					w/ trace gravel			
15	5	CA	80/6"	100% (1st Ret)		7100		
					V. dense, moist, Lt gray w/ reddish brn mottles silty med to fine sand Torrey Sandstone?			
20	6	SP	100/6"	50%		7100		

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-181	SR56-UAR	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

2 of 3

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 77.7 M	BORING NO.: UAR-HSA-2
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	B -

Depth	Sample #	Type	Blow Ct.	Recovery	DESCRIPTION	
	6	SP	100/6"	50%		7100
	7	SAH				
25	8	CA	70/6"	100% (1 Set Ret)		7100
					cemented Sand layer	
30	9	SP	50/6" 70/6"			7100
35					cemented sand layer	7100
	10	CA	58/6"	100% (1 Set Ret)	V. dense, moist, mottled olive brn and reddish brn very silty fine sand (SM), locally fine sandy silt (ML) and dark gray color zones. Delmar Fm	
					Torrey Sandstone	
40	11	SP	70/6"	100%	V. dense, moist, lt gray silty med to fine sand (SM) locally w/ reddish brn mottles	7100

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-181	5R56-UAR	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG 3 of 3

LOGGED BY: J. Brown	DATE DRILLED: 1-11-99	BORING ELEVATION: 77.7M	BORING NO.: UAR-HSA-2
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140lb DROP: 30in	B-

Depth	Sample #	Type	Blow Ct	Recovery	DESCRIPTION				
45	12	ch	90/6"	100% (1st Ret)	w/ increased moisture	71.00			
					BH. 45½' no groundwater				
10									
15									
20									

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I-181	SR56-UAR	FIGURE NO.:

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Engineers and Geologists